VOLUME 81 SEPARATE No. 624

PROCEEDINGS

AMERICAN SOCIETY OF CIVIL ENGINEERS

FEBRUARY, 1955

DISCUSSION OF PROCEEDINGS - SEPARATES

361, 381, 466, 482, 483, 490

HYDRAULICS DIVISION

Headquarters of the Society
33 W. 39th St.
New York 18, N. Y.

PRICE $0.50 PER COPY

Current discussion of papers sponsored by the Hydraulics Division is presented as follows:

This paper was published at 1745 S. State Street, Ann Arbor, Mich., by the American Society of Civil Engineers. Editorial and General Offices are at 33 West Thirty-ninth Street, New York 18, N.Y.

Discussion of
"PRESSURE SURGE CONTROL AT TRACY PUMPING PLANT"

by John Parmakian
(Proc. Sep. 361)

JOHN PARMAKIAN,[1] M. ASCE.—The discussion of Mr. C. E. Withers and Mr. P. Linton have emphasized the ever-present question confronting pumping plant designers: What will be the operating characteristics of the pumps to be furnished for this installation? Mr. Withers points out that the characteristic pump diagrams are not available for most pumps, especially in the zones of "energy dissipation" and "turbine operation." Therefore, it is necessary to compare the characteristics of the pump studied in the region of normal operation with available diagrams. This can be done by the use of homologous relations and the pump performance curves supplied by the manufacturer. In most cases a characteristics diagram is not constructed. Instead, the complete characteristics diagram used is one for the same type pump of about the same specific speed as the pump being studied. Experience has shown this method to be satisfactory for most engineering purposes.

In the study on Tracy Pumping Plant the characteristics in the normal region of operation, as determined by homologous relations, agreed with that given in an article by Professor R. T. Knapp. Therefore, the complete characteristics diagram from the article by Professor Knapp was used. Since it is likely that the characteristics in the region of point A3.5 could vary from one machine to the next, as suggested by Mr. Linton, this could be a possible reason for the difference between the test and calculated values.

It is perhaps unfortunate that the extrapolated point A3.5 is the lowest point and represents the minimum head. The error which may have been introduced by extrapolation would have been reduced if the minimum head had occurred at some later time. Any error introduced at point A3.5 has very little effect on the value of the maximum computed head. The minor difference between the test and computed minimum value is probably due to the use of a characteristic pump diagram which is not truly representative of the pumps. The use of the equivalent line had little effect on the calculated values, because the L/A of the wye branches are small when compared to the total L/A of the line.

Another possible reason for the difference between the test and computed minimum head could be due to the operation of the siphon outlet. The vacuum breaker valve on the siphon outlet pops open when the power is interrupted. This action increases the head at the discharge end of the line as much as 15 feet. This would tend to give a higher minimum head than was calculated as the effect of the siphon was neglected in the calculated solution.

The minor difference between the calculated and observed maximum head is believed to be due primarily to the draining of the line away from the siphon. It appears that during the time of the transient, the level at the

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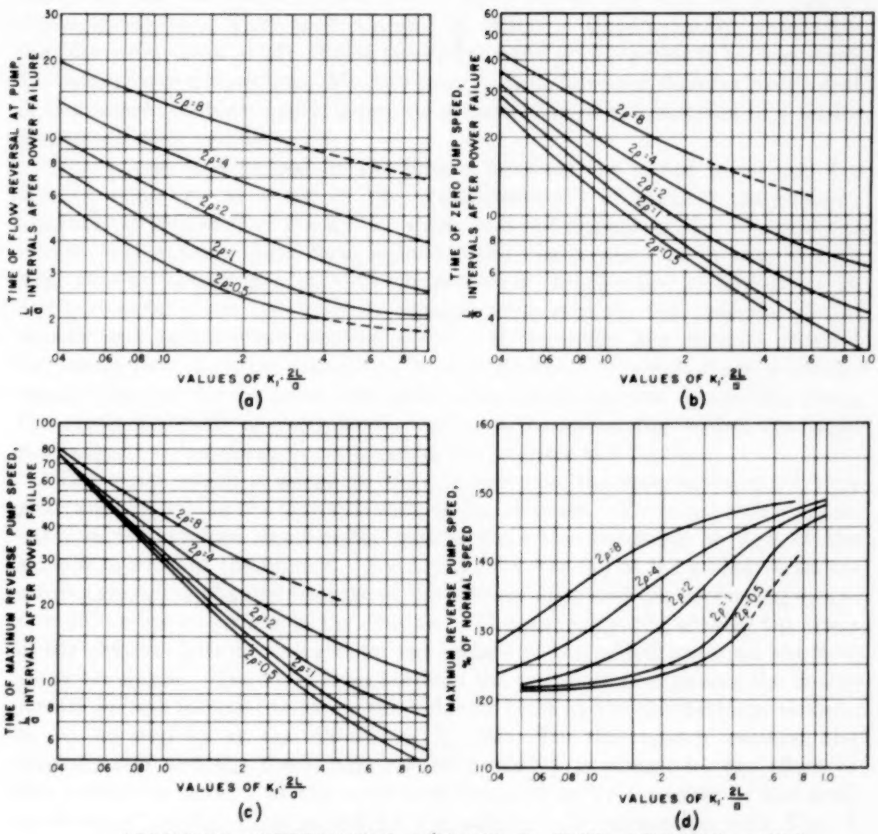
discharge end of the line might have dropped 15 to 20 feet or more. This would lower the maximum head below that computed. However, the head rise probably is very little affected by the pump characteristics since the valve does most of the controlling in the region of negative flow.

Mr. Linton suggests that the difference in head between calculated and test be used as an additional factor of safety if the calculated values can be depended on to be always conservative. This cannot, however, always be depended on, because there are so many parameters whose true values are unknown.

Mr. Linton asks as to the type of instruments used. The instruments consisted of Statham Pressure transmitters, brush strain gauge amplifiers and direct writing oscillographs. The instruments can follow the pressure oscillations very accurately up to a frequency of 100 cycles per second and were adjusted to read pressure changes with an accuracy of less than a foot of head.

Mr. Withers states that the head rise at the pump would have been greater if there was no valve in the line or if the valve had closed very slowly. However, it is also possible to have a higher head rise due to a valve closing too fast or at a rate so that its fast rate of closure occurs at a time when reverse flow takes place. It is also possible that a valve properly operated to keep the head rise to a minimum when two or more pumps fail might give a higher head rise when only one pump fails.

Mr. Withers asks for curves from which the maximum reverse speed and the time history of the transient hydraulic conditions can be determined. These are presented in Figure 15.



ADDITIONAL WATERHAMMER EFFECTS IN PUMP DISCHARGE LINES
DUE TO A POWER FAILURE AT THE PUMP MOTORS

FIGURE 15

Discussion of
"OPEN CHANNELS WITH NONUNIFORM DISCHARGE"

by Wen-Hsiung Li
(Proc. Sep. 381)

TURGUT SARPKAYA,[3] J. M. ASCE.—Flow in open channels with nonuniform discharge has been studied extensively by investigators such as H. Favre,[4] Julian Hinds,[2] and Thomas R. Camp,[5] M. ASCE. Unfortunately, except for the pioneering work of Mr. Hinds, their important works appear to have escaped the attention of the author. Mr. Li's contribution, however, does revive interest in this elusive problem and is a concise presentation of the material in a useful and practicable manner.

The energy loss caused by the impact between the added water and the flowing water is (as noted in the "Introduction") of primary importance. However, the impact of the added water does not appear in Mr. Li's analysis except that a momentum term is introduced—as it was also in Mr. Favre's analysis—by assuming a velocity component in the direction of flow (v'). In the horizontal plane this velocity component is zero for the majority of hydraulic and sanitary engineering works. If, however, the channel receives the water from another channel or from a reservoir in which there is an approach velocity (as for some dam spillways), the component of velocity along the channel should be considered. In such instances, the added discharge may also be nonuniform—complicating the problem still further.

The water which is added to the channel over the weir entrains air, thus generally disturbing the water flowing in the channel. The total resistance to the flow is therefore significantly greater than the resistance to flow at the same rate without the impact. Evidently, the free fall of the added water increases along the channel because of the bottom slope and the decreasing elevation of the water surface in the conduit. Consequently, the greater the slope of the channel bottom, the greater the impact of the added water on the flow along the flume. The drawdown between the upstream section and the downstream section is equal to twice the velocity head at the downstream section, as can be seen by an examination of Eq. 9*b*. The discrepancy between the experimental and the theoretical results probably is caused by the effect of this increasing impact on the resistance rather than by a neglect of the term involving v' in Eq. 1, as stated by the author. In connection with Eqs. 1 and 2, Mr. Li considered the shearing force at the channel walls to be balanced by the momentum of the added water. As has been shown[5] by Mr. Camp, the frictional loss is of little importance in short channels and need not be considered in most cases.

The studies of the nonuniform runoff resulting from rainfall on impervious surfaces are relevant to the present problem. In several studies,[6,7] runoff from a given drainage area was observed to increase markedly just after the

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[4] "Contribution à l'Étude des Eaux Courants," by H. Favre, Rascher & Cie., Zurich, Leipzig, and Stuttgart, 1933.

[5] "Lateral Spillway Channels," by Thomas R. Camp, *Transactions*, ASCE, Vol. 105, 1940, pp. 606–617.

[6] "Miscellanées," *La Houille Blanche*, No. 4, 1950, pp. 498–500.

[7] *Ibid.*, No. 1, 1951, pp. 72–75.

rain ceased and before the runoff decreased gradually to zero. This apparent paradox results from the established fact that the resistance to flow during rainfall is significantly greater than the resistance to flow at the same rate of runoff after rainfall has ceased. Consequently, the surface detention in the equilibrium stage during rainfall is greater than that necessary to maintain the same rate of total runoff after the rain has ceased. The increase in runoff is thus the discharge of the excessive amount of water in detention.

The author's introduction of the dimensionless parameter, G, makes possible a more significant presentation than those made by previous investigators. The parameters used by Mr. Li, however, could have been obtained by a dimensional analysis of the problem in the following form:

$$\frac{y_u}{y_o} = f(\mathbf{F}, G) \qquad (43)$$

For various types of cross sections, Eq. 43 has been presented graphically (Figs. 3 and 6). Although Mr. Li considered channels of various cross sections, he included only channels of constant width. This unnecessary restriction, particularly for large structures, makes impossible the achievement of economy in construction material. If both the width of the channel and the slope of the weir crest are varied, an increase in capacity and a resulting economy in material can be obtained.

Because the discharge is proportional to the $\frac{3}{2}$ power of the head on the weir, the equation of momentum (using the notation shown in Fig. 9) can be written as

$$- \rho\, g\, (\beta\, x + b_u)\, y\, \Delta y - \rho\, g\, \beta \frac{y^2}{2} \Delta x + \rho\, g\, y\, (\beta\, x + b_u)\, S_o\, \Delta x$$

$$- \rho\, g\, f \frac{Q^2{}_x}{A^2{}_x} \frac{1}{8\, g} \frac{b_x\, y}{R_x} \Delta x = 2\, \delta\, \gamma \frac{[(\epsilon\, x + H_u)^{5/2} - H^{5/2}{}_u] H^{1/2}{}_x}{y (\beta\, x + b_u)}$$

$$\times \left(1 - \frac{\Delta y}{y} - \frac{\beta\, \Delta x}{\beta\, x + b_u}\right) \Delta x \qquad (44)$$

in which f is the Darcy-Weisbach resistance coefficient, R_x denotes the hydraulic radius of the cross section,

$$\beta = \frac{b_o - b_u}{L} \qquad (45a)$$

$$\gamma = \frac{2}{3} C \sqrt{2\, g} \qquad (45b)$$

$$\delta = \frac{2}{5} \frac{\gamma}{\epsilon} \qquad (45c)$$

and

$$\epsilon\, \alpha = \frac{H_o - H_u}{L} \qquad (45d)$$

The existing relationships are

$$H_x = \epsilon x + H_u \quad \text{(46a)}$$

$$b_x = \beta x + b_u \quad \text{(46b)}$$

$$A_x = y(\beta x + b_u) \quad \text{(46c)}$$

$$R = \frac{y(\beta x + b_u)}{2y + \beta x + b_u} \quad \text{(46d)}$$

$$q = \Delta Q = \gamma H^{3/2}_x \quad \text{(46e)}$$

$$Q_x = \gamma \int_0^x (\epsilon x + H_u)^{3/2}_{dx} = \delta[(\epsilon x + H_u)^{5/2} - H^{5/2}_u] \quad \text{(46f)}$$

and

$$Q_o = \delta(H^{5/2}_o - H^{5/2}_u) \quad \text{(46g)}$$

The surface profile can be obtained from a known depth of flow by numerical integration. In this manner the free fall of the nappe near the outlet is de-

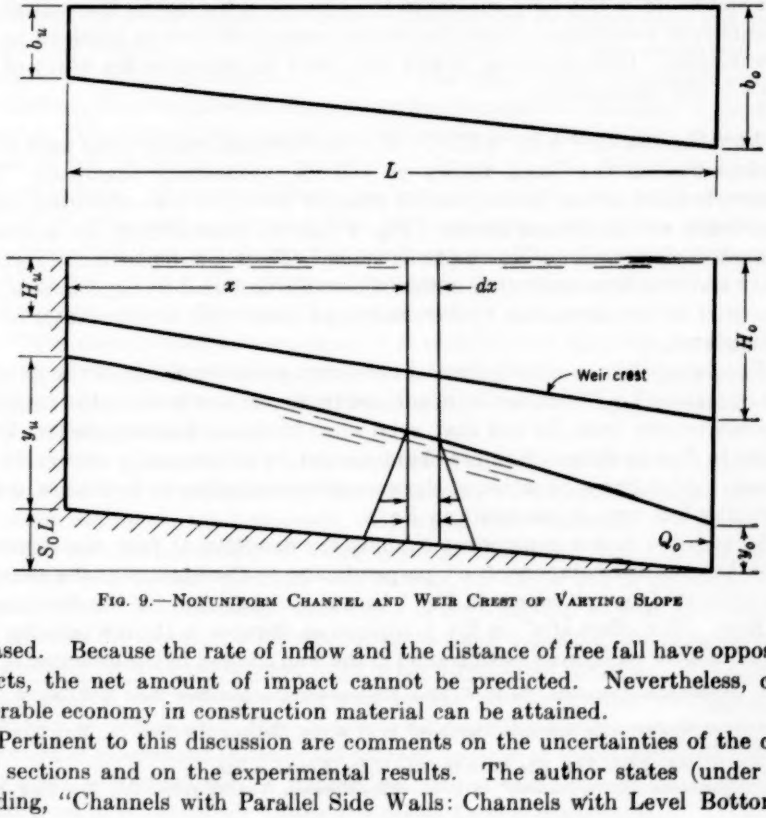

FIG. 9.—NONUNIFORM CHANNEL AND WEIR CREST OF VARYING SLOPE

creased. Because the rate of inflow and the distance of free fall have opposing effects, the net amount of impact cannot be predicted. Nevertheless, considerable economy in construction material can be attained.

Pertinent to this discussion are comments on the uncertainties of the control sections and on the experimental results. The author states (under the heading, "Channels with Parallel Side Walls: Channels with Level Bottom")

that:

"When free fall occurs at the outlet end, the flow is critical, and therefore, $\mathbf{F}_o = 1$. The depth y_o can be computed through the knowledge of Q_o."

This is not quite precise; actually, the depth at the rim of the fall is considerably less than the critical depth ($0.655\, y_c$ to $0.715\, y_c$).[8,9] The distance between the location of the critical depth and the rim of the fall has been found[8] to be approximately $11.6\, y_c$. However, the uncertainty of the control sections should be remembered—not only in the case of a free fall—but also in all the other cases cited by Mr. Li.

The experiments made by the author to demonstrate the validity of his results are remarkable. Unfortunately, no mention was made as to the method whereby this highly disturbed, wavy, air-entrained water surface was measured in order to obtain such precise results. It is stated (under the heading, "Channels with Parallel Side Walls: Channels with Level Bottom"):

"Thus, the corresponding observed and computed values are in agreement. The application of the momentum equation to this case is therefore acceptable."

This statement is not fully substantiated because of the lack of theoretical and experimental knowledge concerning several important factors involved in the phenomenon. Compensating factors may tend to minimize the effect of the errors in the assumptions.

Wen-Hsiung Li,[10] A.M. ASCE.—The fundamental relationship used in the development of the basic theory is Euler's momentum theorem.[11] This theorem is exact as long as it is possible to apply Newton's laws of motion and to treat fluids as continuous media. Eq. 1 follows immediately for prismatic channels from the several assumptions made. These assumptions are justified by the fact that the results thus obtained are substantiated by experiments; the main part of the discussion by Mr. Sarpkaya deals with the exactness of the assumptions.

The energy loss caused by impact does not appear explicitly in the momentum equation (Eq. 1) but has been included in the various terms in the equation. This can be seen from the fact that, even when frictional loss is neglected, Eq. 1 results in flow systems which are characterized by a decrease in energy downstream. As in the application of the momentum equation to hydraulic jumps, no energy-loss term is necessary in Eq. 1.

In addition to the component v' along the direction of flow, the incoming water has another component, v'', perpendicular to the bottom of the channel. The term v'' does not appear in Eq. 1 because it is written for the direction of the flow. The effect of v'' on Eq. 1 appears as changes in (1) the velocity distribution over each cross section and (2) the wall friction in the direction of the

[8] "Handbook of Hydraulics," by H. W. King, McGraw-Hill Co., Inc., New York, N. Y., 3d Ed., 1939, pp. 389–390.

[9] "Fluid Mechanics for Hydraulic Engineers," by H. Rouse, McGraw-Hill Book Co., Inc., New York, N. Y., 1938, p. 323.

[10] Asst. Prof. of Civ. Eng., The Johns Hopkins Univ., Baltimore, Md.

[11] "Theoretical Hydrodynamics," by L. M. Milne-Thomson, The Macmillan Co., New York, N. Y., 1950, p. 74.

flow. Such changes can be accounted for in Eq. 1 by using the proper velocity distribution and frictional coefficient, which, unfortunately, are unknown (as of 1954). By making assumptions 2 and 4 (under the heading, "The Momentum Equation") in obtaining Eq. 1, it was implied that the effect of v'' is only secondary in the flow systems that were cited. That these assumptions are justified can be seen from the test results listed in Table 1. In these tests, the magnitude of v'', although varying along the channel in each test and varying also from test to test, did not affect the validity of Eq. 9. This investigation of the effect of v'' is interesting but does not affect the validity of the results presented; they have been obtained by neglecting the frictional resistance. That this resistance can be neglected in practical cases has been demonstrated quantitatively in Fig. 8.

The critical depth is the depth of flow with minimum specific energy which has physical meaning only when the pressure distribution is hydrostatic. In the analysis of flow in open channels, the pressure distribution, for simplicity, is usually assumed to be hydrostatic with the inherent result that the depth at a free drop is equal to the critical depth. The well-known discrepancy between this result and the experimental observations is caused by the curvature of the water surface which causes the pressure distribution to depart from the hydrostatic distribution. This effect of surface curvature has been mentioned in assumption 3 in the derivation of Eq. 1. The departure of the depth at a free outlet from the critical depth depends on the flow and the geometry of the structure; in this problem the depth and geometry are different from those which occur in flat channels with uniform flow, for which Mr. Sarpkaya has cited several values. The critical depth has been used for the free outlet of channels with $G \leq 2$ in order to be consistent with assumption 3. When applied, this approximation does not change the result appreciably. For cases with critical depth in the channel, the surface curvature at the critical section is very small, and the error caused by using this assumption is always negligible.

These assumptions were made so as to obtain practicable results for a large variety of channel sections in terms of three simple dimensionless parameters. Without these justifiable assumptions, these practicable, simple results would not have been possible.

Prismatic channels with uniform bottom slope and uniform addition of water have been investigated. These channels were chosen because they are the most commonly used channels. By citing these channels, the writer does not rule out the possibility of other types of channels. Eq. 1 is completely applicable to prismatic channels with nonuniform slope and nonuniform addition of water. Mr. Sarpkaya uses Eq. 1 in deriving Eq. 44 with the omission of the term involving v'. He also seems to have neglected the component of the pressure force on the tapered walls in the direction of the flow. The channel with a sloping weir crest (as suggested by Mr. Sarpkaya) may result in a less expensive structure, but the economic advisability of the arrangement for spillways is questioned because the storage volume of the reservoir below the weir crest is reduced with a sloping weir crest—even though the same high water level in the reservoir during the design flood is maintained.

In the experiments reported by the writer, the water-surface elevation was measured with a point gage. Each of the depths listed in Table 1 and Table 2 is the average of ten readings. In these experiments, a phenomenon of practical interest was observed which the writer did not cite in order to maintain continuity in the presentation. It was observed that the weir can be drowned to a depth of approximately $H/2$ without decreasing its capacity. Thus, the required depth of channel at the upstream end is only $(y_u - \frac{1}{2} H)$. A margin of safety will be provided if a depth equal to y_u is used.

One might question the method in which Eq. 43 was obtained through dimensional analysis of a problem involving so many variables. Eq. 43 should involve the dimensionless parameter B. Without examining the differential equations and their results, the advisabiltiy of using the parameters F_o, G, and B for various cross-sectional shapes could not have been ascertained. Because the functional relationship must be obtained by integrating the differential equations, dimensional analysis is not useful in this case.

To Mr Sarpkaya's list of previous investigators one might add C. N. Miller[12,13] and M. F. Stein.[14]

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[12] "An Approximate Formula for Calculating the Discharge Capacity of Rapid Sand Filter Wash Water Troughs," by C. N. Miller, Appendix B, in "Water Purification," by J. W. Ellm, McGraw-Hill Book Co., Inc. New York, N. Y., 1928.

[13] "Diagram for the Design of Wash Water Gutters," by C. N. Miller, *Engineering News-Record*, Vol. 90, 1923, p. 882.

[14] "The Design of Wash Water Troughs for Rapid Sand Filters," by M. F. Stein, *Journal*, A.W.W.A. Vol. 13, 1925, p. 411.

Discussion of "FLOW IN OPEN CHANNELS"

by Edward F. Wilsey
(Proc. Sep. 466)

ARTHUR TOCH,[1] J.M. ASCE, and EMMETT M. LAURSEN,[2] A.M. ASCE.— To substantiate the opinion that Mr. Wilsey's paper contains many technically incorrect statements, the following is presented:

1) With respect to Eq. 1, no "drag formula" will yield the Bernoulli equation, which is based on Euler's equations of motion. The Bernoulli equation, furthermore, is applicable to nonuniform flow as well as to uniform flow.

2) All the material presented, such as Table 7, can be found in far more complete form in standard reference texts on fluid mechanics.

3) The relationships presented are and have been utilized as a basis for measuring devices such as the "Parshall" flume.

4) Nothing has been demonstrated by the use of the terms "tranquil" and "rapid or shooting" as synonyms for subcritical and supercritical in reference to open-channel flow.

5) No reason can be given for the statement (under the heading, "Two Types of Flow in Open Channels") that two different formulas must be used for subcritical flow and supercritical flow; supercritical flow is not necessarily aerated flow.[3]

6) Is not the dimensional analysis (under the heading, "Two Types of Flow in Open Channels: The Dimensional Analysis") superfluous?

7) If Eq. 10a denotes a Taylor expansion, the exponents would be fixed as a series 0, 1, 2, ... , n.

8) The usual application of the π-theorem for the variables V, g, J, and ν will result in Eq. 12c in a more generally understandable manner. Of course, the slope and a roughness length such as k, and possibly other variables affecting the flow, should be included in the analysis.

9) Even though the Chezy equation is desired, one should not arbitrarily substitute R for J or suddenly include the slope, sin θ. The Chezy equation cannot be derived from dimensional analysis alone but requires the additional use of a free-body diagram.[4]

10) From a proper dimensional analysis one would obtain the coefficient B (Eq. 14a) as a function of a Reynolds number, a relative roughness, a shape factor, and other parameters.

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2. Research Engr., Iowa Inst. of Hydr. Research, State Univ. of Iowa, Iowa City, Iowa.
3. "High-Velocity Flow in Open Channels: A Symposium," Transactions, ASCE, Vol. 116, 1951, p. 265.
4. "Fluid Mechanics for Hydraulic Engineers," by Hunter Rouse, McGraw-Hill Book Co., Inc., New York, N.Y., 1938, pp. 276-279.

11) The introduction of aerated flow (under the heading, "The Experimental Evidence") is unrelated to the previous work and proves nothing.

12) The statement regarding the relationship (Eq. 17) between water portions and air portions of the flow is indeed correct—in fact, it is tautological. The statement regarding the resistance coefficient, B, is open to question.

13) Research[5] on air entrainment has gone considerably deeper into the causes and effects than can be ascertained from Mr. Wilsey's presentation.

14) Eq. 24 should probably contain a constant, but it is otherwise correct for a single vortex.

15) Eq. 28a might be improved by replacing the first equal sign by a plus sign. However, this would still leave the term, $K\ p_a$ without significance.

16) The idea presented (under the heading, "The Stream Function") that "* * * an eddy at the surface will draw in air at its center * * *" is a novel one and would have to be reinforced by considerable experimental evidence before being believed. Research[5] has developed a concept for air-entrainment phenomena in a more believable manner.

17) The significance of the temperature measurements (under the heading, "Flow in Open Channels of Low Slope") is completely unexplained. It can be assumed, of course, that this is meant to refer to the earlier mention of viscosity, but a connective statement is completely lacking.

18) It is not indicated which dimensional equation was reduced to Eq. 32. It would also be of help if the term d_c were defined more clearly because, according to the Appendix, d_c is a drag force of some kind. Most probably, y_c should be used instead of d_c both in Eq. 32 and in Eq. 33a. If d is replaced by y in Eq. 33a, and if Eq. 30 is used, Eq. 33b can be obtained except for the constant. Similarly, if in Eq. 33c, y_a which is not defined at all is assumed to be a mean depth of flow such that

$$Q = V\ y_a\ b \tag{35}$$

Eq. 33c can be obtained from Eq. 33b, but the constant is still incorrect. It is not explained where the numerical value of 1.44 was obtained for Eq. 33a. The superiority of Eqs. 33 over the Manning of Darcy-Weisbach evaluation of Chez y's C has certainly not been demonstrated even for the limited data cited.

19) Most of the omissions of items in Table 7 seem to have been caused by lack of some data in the original tabulation, which is quite reasonable. A few omissions, however, cannot be so explained. Great space could have been saved by reprinting only the reference numbers, the mean velocities, and Manning's coefficients presented in the original tables, with the corresponding coefficient of roughness from Eqs. 33.

20) The condition that certain values of the roughness coefficients should be repeated is met both by Manning's n and by the n in Eqs. 33. Manning's n, furthermore, does not decrease with decreasing velocity as stated (nor does it consistently increase in that way), as can be seen from a most cursory examination of the data in Table 7.

21) Using Eq. 30 and the mean depth y_a, Eq. 34 can be rewritten simply as

$$V = K \left(\frac{V\ y_a}{\nu} \right)^{2/15} \sqrt{2g\ R\ \sin\theta} \tag{36}$$

if the radius vector r is eliminated as it should be and if d_c is assumed to be

5. Proceedings, Minnesota International Hydraulics Convention, IAHR-ASCE, Minneapolis, Minn., 1953.

y_c. This, however, leaves one to ponder over the meaning ascribed to K in the Appendix and to question whether temperature affects K as well as ν.

The foregoing items do not constitute a complete criticism. A large number of the statements made by Mr. Wilsey are made in such a way that a discussion of them would entail a great deal of expository writing on the subject of fluid mechanics and of logic and science in general.

Discussion of
"EQUATION OF THE FREE-FALLING NAPPE"

by Fred W. Blaisdell
(Proc. Sep. 482)

WALTER RAND,[1] A.M. ASCE.—The paper of Mr. Fred W. Blaisdell is a valuable contribution to the hydraulics of free-falling nappe and should be welcomed by anyone concerned with the design of straight drop spillways and stilling basins. However, to ensure the correct application of the equations given in this paper by the designers, some other aspects of the overfall phenomena deserve attention.

It is known that the nappe is not free-falling over all the distance from the weir crest to the apron but that there is a water cushion between the nappe and the vertical wall of the weir. In the case of the non-submerged nappe, the waterlevel under the nappe is considerably higher than it is immediately downstream. Supported by the watercushion, the nappe changes its curvature and turns smoothly into horizontal, supercritical flow on the apron. Of course, the equations for the free-falling nappe are valid only over the distance between the weir crest and the water surface back of the nappe. This sets a higher limit to y and y/H because the highest value y_{max}, corresponding to the distance between the weir crest and free surface back of the nappe, is smaller than the total height h of the weir.

The depth of the watercushion back of the nappe has been investigated by Dr. Walter L. Moore[2] and by the writer (unpublished) for the case of subcritical approach velocities and for the approach channels with the bed level at the weir crest. The distance y_{max} for the free-falling nappe will equal the total height h of the weir only for H = 0 and will be zero at approximately H/h = 1.7. No air filled space will be left under the nappe in the latter case. The distance y_{max} for an average case of H/h = 0.3 will be equal to 0.63 h, to give an idea of the variation of H/h vs. y_{max}/h.

It follows from what was said that the distance x_{max} from the weir, at which the nappe is supposed to hit the apron, has little physical significance if it is computed from the equations of a free-falling nappe. Of course, the position x_{max} can always be useful as a reference point for the designers of drop structures.

Another significant position would be located at the distance d from the weir at which the tangent to the non-submerged upper nappe will become parallel to the apron. This marks the initial point of flow in the downstream channel. The mean velocity here is parallel to the surface of the apron and the velocity is the maximum reached in the overfall. This point could also serve as the reference point from which to measure the distance for a proper location of sills for energy dissipation. The writer gives here some illustrative values of the distance d from his measurements to be compared with

1. Asst. Prof. of Fluid Mechanics, New York Univ., New York, N.Y.
2. "Energy Loss at the Base of Free Overfall," by Walter L. Moore, Transactions, ASCE, Vol. 108 (1943), p. 1343.

x_{max} of the lower nappe. For example, in the case of $h/H = 2$ the ratio d/x_{max} is 1.36. Only for $h/H \geq 8$, that corresponds to the depth of the water-cushion under the nappe smaller than 0.18 h, is the difference between d and x_{max} negligible. However, the smaller straight drop spillways operate in most cases at values $h/H < 8$.

The writer hopes that the remarks made above will help the reader understand the extent of the approximation made if the free-falling nappe equation is used for the lower part of the overfall where it is not free-falling.

JOHN C. HARROLD,[3] A.M. ASCE.—Mr. Blaisdell has performed a valuable service to the profession by bringing together into one place all of the important experimental data on the shape of the free over-falling nappe and by analysing these data. The list of references in his paper will provide a source of data for other engineers who wish to make similar analyses for other purposes. Mr. Blaisdell's equations and coefficients will be very useful to engineers having drop structures and over-fall spillways to design.

In 1941 the Office, Chief of Engineers, Department of the Army, undertook a similar analysis, utilizing all of the important experimental data which had been published up to that time, for the purpose of determining a standard overflow crest which could be used on all Corps of Engineers' high concrete gravity spillways. Lower nappe data from the following experimenters were plotted on a large-scale graph down to a $\frac{y}{H_w}$ value of about 1.8:

Bazin: $\frac{h_v}{H} = 0.0025$

Scimemi: $\frac{h_v}{H} = 0.0028$

Winter (Alabama Power Co.): $\frac{h_v}{H} = 0.0036$

Bur. of Recl. (Fort Collins Tests): $\frac{h_v}{H} = 0.0001$ to 0.002

Bur. of Recl. (Denver Tests): $\frac{h_v}{H} = 0.002$

Creager and Justin (lower nappe)

Values of $\frac{y}{H_w}$ greater than 1.8 were not considered pertinent to the design of overflow spillway crests since the slope of the lower nappe at this point exceeds the slope of the downstream face of an ordinary concrete gravity spillway. The term, "H_w", as used above, is the head on the weir measured from the water-surface upstream from the weir to the level of the sharp-crested weir and does not include the velocity-of-approach head.

After reading a description of all of the above tests, it was decided that the Bureau of Reclamation's Denver tests (Author's reference (2), p. 64, Table 12) were the most accurate. A curve was therefore fitted to the Denver data. The high point (crest) of the lower nappe was found to occur at a value of $\frac{x}{H_w}$ of 0.250 and $\frac{y}{H_w}$ of 0.112. The portion of the crest upstream from this crest was fitted by a compound curve with a radius of 0.444 H_w from the high point of the crest to a point 0.155 H_w from the crest and a radius of

3. Hydr. Engr., Office, Chf. of Engrs., U.S. Dept. of the Army, Washington, D.C.

0.178 H_w from this point to the sharp-crested weir. This curve is horizontal at the crest. The portion of the curve downstream from the crest was fitted by an exponential curve with the following equation:

$$\left(\frac{x}{H_w}\right)^{1.85} = 1.809\left(\frac{y}{H_w}\right) \qquad (1)$$

The origin of this curve is at the high point of the lower nappe, as defined above, and will hereafter be called the "crest" or "spillway crest."

Since the design of an overflow spillway is usually based on the head above the crest, the above equations and distances were rewritten in terms of H_c, the head on the crest. "H_c" does not include velocity of approach. Since the high point of the lower nappe was found to be 0.112 H_w above the sharp-crested weir, the following relation was used to transform the above equations and distances into H_c values:

$$H_c = 0.888\ H_w \qquad (2)$$

The following equations and distances, therefore, define the standard spillway crest which has been used by the Corps of Engineers for high concrete gravity spillways since 1944:

Horizontal distance from upstream face of dam to spillway crest = 0.282 H_c

Vertical distance from upstream end of crest curve to spillway crest = 0.126 H_c

Compound curve upstream from crest:

R = 0.5 H_c (from crest to point 0.175 H_c upstream)

R = 0.2 H_c (from above point to upstream face of dam)

Exponential curve downstream from crest (origin at crest of spillway):

$$\left(\frac{x}{H_c}\right)^{1.85} = 2\left(\frac{y}{H_c}\right) \qquad (3)$$

Equation (3) is ordinarily used in the following form (origin at crest of spillway):

$$x^{1.85} = 2H_c^{0.85}y \qquad (4)$$

If a quarter of an ellipse is preferred to a compound curve for the upstream portion of the crest, the following ellipse will pass through the experimental points:

$$\frac{x^2}{(0.282H_c)^2} + \frac{y^2}{(0.150H_c)^2} = 1 \qquad (5)$$

The origin of this ellipse is a distance 0.150 H_c vertically below the spillway crest.

The writer plotted the above standard crest on the same graph as the lower nappe curve shown on Figure 4, for comparison. The curves coincide from about $\frac{x}{H_w}$ = 0.5 to about $\frac{x}{H_w}$ = 2. Below $\frac{x}{H_w}$ = 2 the two curves gradually separate, the author's curve falling below the standard crest. At $\frac{y}{H_w}$ = 7, the horizontal difference is about 0.01 H_w; at $\frac{y}{H_w}$ = 11, it is about 0.02 H_w. It will be seen that the difference is small, even though the

standard crest curve was developed only from data above $\frac{y}{H_w} = 1.8$. It was also found that the author's curve on Figure 4 could be approximated by the exponential curve:

$$\left(\frac{x}{H_w}\right)^{1.92} = 1.882\left(\frac{y}{H_w}\right) \tag{6}$$

This equation in terms of H_c is as follows:

$$\left(\frac{x}{H_c}\right)^{1.92} = 2.1\left(\frac{y}{H_c}\right) \tag{7}$$

The origin of equations (6) and (7) is at the crest, as in the case of equations (1) and (3).

The above standard crest was checked by model experiments to determine pressures on the crest and discharge coefficients for various heads. The tests were conducted at the Waterways Experiment Station of the Corps of Engineers located at Vicksburg, Mississippi. Tests were conducted both with and without piers on the crest. The discharge coefficient[4] without piers was found to be 4.03 with the design head, H_c, on the crest. Pressures were slightly positive for this condition.

The philosophy behind the adoption of the above standard crest was that this crest would give the maximum discharge possible without any negative pressures on the crest. With no negative pressures on the crest, there would be no tendency for the jet to spring free from the crest, nor would there be any negative pressures to consider in the stability of the cross-section.

In the past few years, however, the Corps of Engineers has adopted a more liberal policy in spillway crest design. Since all past model experiments have shown that the overflowing nappe will cling to the crest until a head considerably larger than design head is reached, it was considered desirable in the interest of economy to permit a maximum head on the spillway somewhat in excess of the head for which the crest is designed. Maximum heads up to 1.33 times the design head of the crest have been permitted, this value being based on model experiments which show that the negative pressure will not exceed about 0.6 H_c with piers on the crest. About 20 feet of negative head is considered the limiting value for this condition. Spillway crests designed for negative pressures have been called "underdesigned crests." In cases where "underdesigned crests" are used, a thorough model study is made to determine pressures on the crest for all discharge conditions so that these pressures can be taken into account in the stability analysis of the spillway section. In some cases, appreciable economies have been effected by the use of an "underdesigned crest"; in others the stability calculations have not permitted it. The coefficient of discharge of the standard crest with a head of 1.33 times the design head on the crest is about 4.13. In addition it has been found that the pier contraction coefficient is slightly less at a head of 1.33 times the design head. The net result is an increase in discharge capacity at the maximum head of about 5 per cent.

Although the standard crest was developed for high overflow spillways with negligible velocity of approach, model tests have shown that this shape can be used for spillways as low as the maximum head on the crest, that is, where the depth of approach below the spillway crest is equal to the maximum depth of water over the crest. Under this condition pressures on the crest

4. Discharge coefficient, C, in the formula, $Q = C(L-KNH)H_e^{3/2}$, where H_e includes velocity of approach head.

are about the same as for a high spillway and the discharge coefficient is only slightly lower, about 4.00.

The standard crest has also been tested for discharge under vertical crest gates and Tainter crest gates located either directly above the crest or a short distance downstream from the crest and pressures have been found satisfactory for these conditions. Pressures on the crest are slightly negative for some gate openings but the negative pressures do not exceed about 0.1 H_c with a head of 1.33 H_c on the crest. This maximum negative pressure occurs adjacent to the piers.

Discussion of
"FLOOD INSURANCE"

by H. Alden Foster
(Proc. Sep. 483)

EDGAR E. FOSTER,[1] M. ASCE.—Flood damage is a big field of risk-taking that has not yet been substantially covered by insurance operations. The author has presented an able analysis of the situation that ought to assist materially in promoting possibility of such insurance.

As pointed out by the author there are several aspects of flood insurance that would require careful study. One of the aspects is the amount of annual flood losses. Various agencies have made estimates for their own purposes but the amount may be only rough approximations of damage sustained. In particular the earlier estimates of losses by the Weather Bureau are very rough. The later estimates since World War II, however, are of appreciably higher standards and probably give reasonable general estimates of the losses sustained for particular floods. Likewise the estimates of the Corps of Engineers are variable. On occasions when adequate funds were available to make a comprehensive survey, results as accurate as attainable were obtained. At other times when only a small crew could be put in the field, results would be only roughly approximate. In general the damage surveys made by the Corps of Engineers are made to justify projects for flood control. For this reason it is not likely that all losses reported would be insurable. For example the "indirect" or "intangible" losses would probably not be insurable.

Another type of loss that does not seem insurable is the Depreciation Losses. The practise of the Corps of Engineers has not been uniform in its consideration of this loss. It was not included as a loss in the damage survey made by the Boston District for the flood of April 1936 because such depreciation of the capital value of real estate following a flood was considered to be a subjective capital evaluation by the public of the annual flood loss or the annual risk of flood damage. This view appears to the writer to be an eminently sound one notwithstanding that various districts of the Corps have included such depreciation with other losses. Following this view such losses should not be insurable because they would be compensated by insurance on other losses.

The author states that the determination of the annual average loss would be one of the basic problems in flood insurance. This is correct, yet it probably would be better to consider the annual flood risk as the basic problem in order to emphasize that the risk may exist in certain places, even without a known flood and exists always in certain areas, even if the last flood is forgotten.

The annual flood risk, like the annual flood loss, is computed as a product of the loss caused, or is causable, by a flood event multiplied by the probability of that event. This makes the probabilities of floods a critical factor in flood insurance.

1. Cons. Engr., Wheatridge, Colo.

Engineers frequently speak of their array of flood data as a sample, but it is doubtful that such an array is a proper sample. Taking a sample implies some selection of data but there is no selection in a full array of flood data. There are a certain number of data in the array and those are all that are available: one uses this array in one manner or another and hopes that it can be applied to an infinitely long record. The only sampling that is done is in the use of the so-called annual flood method where the maximum annual flood is taken for an array. This is certainly taking a sample with a bias or prejudice, which is contrary to good sampling practise.

In the writer's opinion the selection of one flood a year, the highest annual flood, is a very undesirable method of handling flood data. In this opinion it is far sounder to use all the available data including as far as practicable all crests caused by distinct meteorological storm events or hydrologic events such as ice jams. Practical aspects would fix a base just high enough to permit selecting distinct crests during the high water seasons; this would cause the rejection of small peaks in the low water season but other than a uniform annual base seems to be an impractical refinement. Naturally many of these small peaks would not cause flood damage. Nevertheless their inclusion in the array assists materially in determining more accurately the mean and the second and third moments for determination of probability. In one example[2] there were 587 flood events above a base stage of 10,000 cfs; these crests were selected to include with reasonable certainty only independent events. Surely the statistical moments were on a much sounder basis by using all these events than they would have been by taking only one flood per year, leaving only 57 events in the series.

The writer still believes that J. J. Slade's[3] partly bounded function is the best yet devised for analysing flood data when the highest accuracy is desired. The applicability of this logarithmic transformation of the normal probability function is indicated by the fact that an array of flood data, including all floods above a low base, plots as a straight line on Hazen's logarithmic-probability paper. This function requires more work in its use than a number of other methods of determining flood frequencies. However, it is not agreed that a criterion of selection of method of calculation of frequencies should be "which method is most convenient to use," except possibly in the most unimportant work.

Furthermore, it seems that the selection of a method for determination of flood frequencies must necessarily be based to a large extent upon theoretical considerations, that is to say, the first criterion of acceptance must be sound theory from both the mathematical and physical viewpoints. The reason for this is that compared with statistical series in other fields, those of flood data contain comparatively few items; even a series with 587 items is small when compared with hundreds of thousands available in the study of human life. Any statistical series is composed of data determined to some degree by chance, so that one series will inevitably differ from another even if both are composed of the same sort of data. One can only expect certain characteristic values, such as the mean and to a diminishing extent the higher moments, to be similar and these moments only if the series is of adequate length. These moments can be used to compute the probability of various

2. Foster, Edgar E., Rainfall and Runoff, New York, The Macmillan Co., page 381.
3. Slade, J. J. Jr., "An Asymmetric Probability Function," Trans. ASCE., page 35.

floods, provided a sound mathematical basis is available.

In order to make flood insurance a paying business, the premium must be established to cover the average annual flood risk plus the loading to provide the insurers' expenses and profit. This apparently will not be such a simple procedure as it is in fire insurance, for example. Each individual property applying for insurance would probably require an inspection to determine its susceptibility to damage, if flooded, and its relationship to the source of flood water and whatever form of flood protection that may have been provided. It is likely that some sort of zoning of areas with approximately equal risk with respect to potential flood heights may be of material benefit in establishing the premium in places where a number of properties are seeking insurance. This inspection should receive some careful engineering study.

In order that one company would avoid losses beyond its capacity to pay in event of a big flood, it would be necessary to scatter its policies over a wide area, taking a few in one city, a few more in another valley, and others in divers places not subject to the same flood. Another means of spreading the risk would be the use of re-insurance.

If flood insurance were generally available, it is probable that various measures would be taken to reduce the frequent losses in low-lying areas. If nothing more were done, it seems that the high premiums necessarily charged for such areas would prove to property owners what high risks that they are incurring by occupying such areas. Some simple measures of protection would be markers to indicate potential flood levels. Additional protective measures could be provided as the economics of the situation justified. These developments would be in line with similar work done by fire and accident insurance organizations.

The author closes his paper with a conclusion quoted from the report of the Flood Committee of the Insurance Executives Association. It appears that this conclusion is unnecessarily pessimistic. Surely the "virtual certainty of loss" cannot be a sound reason for lack of flood insurance: life insurance is extensively practised in spite of the fact that loss at death is absolutely certain, the only uncertainty in this matter is the time of occurrence. The fire losses of the country would also be catastrophic if they all occurred simultaneously in one city; but since they are scattered in time and place, they do not prevent the furnishing of insurance against fire loss. As the matter now stands with respect to flood damage, someone carries the flood risk and, since flood insurance is not available, it is the property owner who carries the risk. Insurance is simply hiring someone else to carry the risk so that a large sudden loss will not cause financial disaster to a property owner; this situation is applicable to property owners in areas subject to inundation. It seems therefore that flood insurance is simply a matter of fixing a recognizable potential loss for individual property and determining a suitable premium for carrying the risk of that loss. As pointed out by the author there are certain problems peculiar to flood risk that must be solved but it is believed that these problems are not insuperable. Apparently flood insurance will have to await a new set of Insurance Executives who can and will meet the problems involved.

C. J. POSEY,[4] M. ASCE.—Insurance of any type, in order to be generally acceptable, has to provide real benefits to everyone concerned, and on a

4. Prof. of Hydraulics and Structural Eng., and Head, Dept. of Civ. Eng., State Univ. of Iowa, Iowa City, Iowa.

reasonably equitable basis. An insurance program of considerable size may be established through the efforts of a special group, but if in time it is found that the design of the program benefits one group more than others who are concerned, there will be insistent demands for its modification. In fact, redesign or modification is almost inevitable during the first years of operation of any new type of insurance, for it is impossible to foresee all the ramifications that will develop where estimated risks may be affected by the vagaries of human nature.

An example is provided by the comparatively recent development of "hospital bill" insurance. The impetus for this program was provided by the hospitals, faced simultaneously with mounting costs and mounting bill delinquencies. The protection offered was often the payment of (nearly) all hospital bills for hospitalization "up to" some stipulated period, often only two weeks. Two defects have come to light: (1) Many persons make a determined effort to collect, each year, more than they pay in premiums; (2) Little or no provision was made for protection of policyholders who require hospitalization for long periods of time. To meet the first defect, premium rates have been increased. This is probably not the real solution, since the class of individuals mentioned will only redouble their efforts to collect, and the others will ultimately find the resultant rates too burdensome.

A rash of lawsuits is evidence of the beginning of public discovery of the second defect, and some companies have already modified their policies to eliminate the upper limit on hospitalization period. Since the cost of this change is not great, it will undoubtedly satisfy the public, hospitals, and insurance companies alike. Few people enjoy extended hospital confinement, and it seems unlikely that the number of lengthy hospital stays will be materially increased by this provision.

Hospital-bill insurance has been discussed in some detail to illustrate that designing a really acceptable insurance program is difficult, and that important changes have already been found necessary, with still more in prospect. It should be noted, too, that it took the organized efforts of the hospitals to put this program across.

At present, no one knows what the principal features of a satisfactory flood insurance program would be, or what organized group could promote its inception. In investigating the possibilities, it is natural to assume that flood insurance would be like fire insurance, or wind-storm insurance, or some other well-established type. But these other types differ greatly with respect to (1) the nature of the hazard and (2) the psychological factors, or patterns of human reaction, so that a careful study of both must be made before the outlines of a satisfactory flood-insurance program will begin to become evident. It seems to the writer that the author has not paid enough attention to the second part of the problem.

For example, the lower limit of payment should be based on a minimum flood discharge or high-water elevation rather than a fixed financial amount. This would require expert engineering evaluation (surely not much more difficult than fire risk evaluation) but would give the owner an incentive to take steps to minimize his loss. This would include not only steps such as the establishment of a dependable system to insure that valuables would be removed in face of flood threats, but also such actions as promoting the construction of remedial works to minimize damages from the lower floods. After remedial works were installed, the policies should be revised to pay damages not for flood above a certain stage, but for floods overtopping the

remedial works. This would insure careful construction and maintenance.

The possibilities of minimizing over-all flood costs by combining flood insurance with remedial or protective works of various kinds have been investigated by Ibrahim M. Mostafa, J. ASCE, who came to the conclusion that under many conditions the inclusion of flood insurance as part of the over-all plan can result in considerable economies.(1)

The author has given a classification of flood losses similar to that given by Barrows, though not as complete.(2) This type of classification is useful to an agency charged with evaluating total flood losses, but for purposes of planning an attack on the flood problem by engineering and financial means it is not nearly as significant as that which divides the total flood damages into local, regional, and national components. Even the most localized floods do some regional and national damage, and there is no question but that in the case of our greatest floods, serious losses are incurred throughout the whole nation. For a flood of given magnitude in a given watershed, the distribution of local, regional, and national losses can be estimated, probably with not much greater uncertainty than the total loss.

The reason the local, regional, and national distribution is of especial significance is that the responsibility for alleviating flood losses has a similar distribution. This responsibility is recognized by our citizens when they make special Red Cross contributions (often for disasters outside of American—and sometimes in amounts greater than would have been sufficient to prevent the disaster) and has also been recognized by the Congress of the United States in formulating the national policy on flood control protection works.

The exact division of responsibility has seldom, if ever, been given an engineering evaluation, being more often based on political compromise. If it could be carefully and thoroughly evaluated, and then decided upon and applied uniformly to every component of a system designed to minimize future flood costs, the possibilities of cooperation and accomplishment would be greatly increased. For example, if the federal government were to assume 100% of the cost of building reservoirs, the regional government 100% of the cost of channel improvements, and the private individuals 100% of the cost of insurance, it would be impossible to achieve a proper balance of the three components, but if the formula were fixed at say 30% federal, 30% regional, and 40% individual for every component of the system, it should not be too difficult to secure agreement upon the balance that would give minimum total cost.

As an example of a situation that is ready-made for flood insurance, let us consider a flood protection district that already has improvements built and paid for by the district, improvements designed to protect against the "largest possible flood." Advances in the science of hydrology during the last 20 years have made it evident that the protection is not really complete, and that there is a chance, albeit a very small one, that great flood damage might occur as a result of a superflood originally not considered possible. The flood insurance premiums to take care of this possibility would not be large, since the probability of the event is very small, indeed. But even a very improbable event may occur next year, hence a very large reserve fund would be necessary. It would seem quite appropriate for the Federal Government to underwrite this reserve fund until the amount collected from premiums became adequate, since the Federal Government did not share in the cost of the original flood protection works. The necessary premiums could be added to the millage collected by the District for upkeep and maintenance without hardship to those benefited.

To put an insurance program into effect for this district would be a task for the district, but it would certainly have a strong case to present to both the insurance companies and the Federal Government.

If we reject flood insurance as impossible except on an outright subsidy or relief basis, we will have been defeated before we have really started. The successful completion of the flood protection works of the Miami Conservancy District provides an example of a case where businessman, engineer, and lawyer refused to yield before obstacles of public prejudice, lack of precedent, and technical difficulty that were fully as great.(3)

REFERENCES

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2. "Floods—Their Hydrology and Control," H. K. Barrows, McGraw-Hill, 1948.
3. "The Miami Conservancy District," A. E. Morgan, McGraw-Hill, 1951.

ROBERT LEE SMITH,[5] A.M. ASCE.—This paper presents a very thorough discussion and analysis of the engineering elements of self-supporting flood insurance. The contribution is most timely for it places in sound perspective a subject that has heretofore received more publicity than thought. Of equal importance is the concept that many of the problems and uncertainties cited by the author are also fundamental to investment principles associated with structural control measures.

Uncertainty over the adequacy of basic data is a vital factor in the conclusions expressed. The lack of accurate information on previous damages suffered at various flood stages is extremely critical to both structural and insurance problems. To be accurate such data must be obtained systematically and continuously. From a practical standpoint collection of such information must remain a governmental function. Creation of uniform collection procedures among the various governmental groups or assignment of that responsibility to a specific agency would enhance materially the engineering and insurance appraisal of flood relief programs.

The author suggests the possibility of establishing a minimum amount of deductible damage which must be carried by the owner. However, such a provision would not, in the writer's opinion, overcome the probable stimulation in use of property covered by such policies. If and when the time comes that insurance can be effectively used as a flood relief measure it should serve to mitigate overdevelopment of flood plain areas. Presumably an effective insurance program could have a zoning influence if the "minimum deductible" clause related to a limiting stage rather than to a specified amount of damage. This concept would be comparable to the establishment of a vertical line at some selected probability on the author's Fig. 7a. The average annual damage subject to insurance coverage would then be computed from damages caused by floods of lesser probability. The ratio of average annual damage to maximum possible loss is thus materially reduced and future flood plain development might well become more cognizant of the relative degrees of risk involved. Practical selection of the limiting probability could be

5. Director, Iowa Natural Resources Council, Des Moines, Iowa.

expected to vary with the nature of the development.

Objections may be raised that the approach just outlined does not provide relief to existing facilities which suffer damage from the floods of greater probability. It is submitted, however, that enterprises located in flood plain areas continue in existence because of certain economic gains associated with that location. Those enterprises by their very existence indicate that they have weighed the risk of the "nuisance flood" occurrence and have found means of overcoming the adversities associated therewith. Yet the existence of such enterprise remains seriously threatened by its inability to evaluate correctly the calculated risk imposed by the floods of lesser probability.

The concluding statement . . . "it appears that an accelerated flood control program supplemented by such relief payments as are necessary on account of flood damage would be more in the interest of the public than a program of so-called 'flood insurance' which would not be self supporting . . ." presents a paradox inasmuch as the overall cost of flood relief could be reduced if that construction program could be combined with a workable flood insurance program.

Figure 1 serves to illustrate the possible contribution of the insurance program. Curve A typifies the functional relationship between construction costs and construction benefits. The maximum improvement where total benefits exceed or equal total costs is designated as point 1. Maximum differential between construction benefits and construction costs occurs at point 2. Sound economic principles would not justify structural expenditures beyond that point.[6]

Curve B indicates the cost of the insurance premium (minus expense loading) required to supplement the construction program if some predetermined upper limit of protection is desired. This upper limit may or may not include protection or coverage against all existing damages.

Curve C represents the combined costs of the insurance and construction programs for the constant condition that 5 units of flood relief benefits be developed. Inasmuch as the insurance program is based on the concept that benefits equal costs over long periods of time it is apparent that the combined program cost must be a minimum where the differential between construction benefits and construction costs is a maximum. Thus if a flood control district could contract for insurance as well as structural relief they could be expected to seek a construction solution corresponding to point 2 on Figure 1.

Further an organized assessment district might be in a position to avail themselves of such a combined program whereas the same concept if left to the option of the individual policy holders would most certainly fail.[7] It has been rightfully concluded that self-supporting flood insurance rates would be expensive, but it must be remembered that they also place in proper perspective the true cost of flood relief which under certain economic and engineering conditions is just as expensive. There is an area of need and the negative conclusions developed should not cause the profession to reject the equal need for continued appraisal of this problem.

6. Hill, R. A., "Multiple Purpose Reservoirs: A Symposium—Summary and Review of Principles," Trans. Amer. Soc. of Civil Engrs., Vol. 115, p. 891, 1950.
7. Mostafa, I. M., "A Policy for Flood Control," unpublished dissertation, State University of Iowa, p. 91, June 1954.

Discussion of
"GROUND WATER IN THE VERMILION RIVER BASIN, LOUISIANA"

by Paul H. Jones
(Proc. Sep. 490)

RAPHAEL G. KAZMANN,[1] A.M. ASCE.—This paper is a model in some respects: it analyzes a complex hydrogeological situation, points to certain problems, primarily the possibility of the ultimate contamination of a major aquifer in southern Louisiana by salt water from the Vermilion River and, by implication, suggests a complete, straightforward solution—construction of a barrier at the mouth of the river to prevent movement of salt water upstream.

It appears, however, that the data presented in support of this thesis can be interpreted in an entirely different manner—and that the alternate interpretation would lead to an entirely different solution to the problem of aquifer-contamination in southwestern Louisiana.

It has been observed that when a river crosses an aquifer outcrop and water can pass freely from the river into the aquifer (and vice-versa), the area of hydraulic contact is an area in which the piezometric surfaces of the aquifer and the river coincide—there exists a potential boundary common to both hydraulic systems. Where the "line of contact" is absent the systems are independent of one another and no flow can occur between them.

Study of Figures 6-11 shows that the water-elevation in the Vermilion River has no particular relationship to the elevation of the piezometric surface of the aquifer. In Figure 8 contours of the piezometric surface cross the river, for most part, at right angles—there is no evidence of a contour common to both hydraulic systems. The transverse profiles (Figures 9 and 11) show that the piezometric surface of the aquifer is apparently independent of the river surface elevation.

The graphs of Figure 12, which place in juxtaposition the salinity of the water of the Vermilion River at Perry and salinity of water from Well VE-75, which is located about one mile south of Perry and 4000 feet from the river, do not necessarily (as the author implies) show cause and effect—or any other simple direct relationship. In the absence of first-hand knowledge of the situation in the area, there are at least two independent lines of reasoning which make it doubtful that the entrance into the aquifer of salt water from the Vermilion River caused the observed changes in salinity of water from the well:

1) The volume of water in storage in an aquifer 200 feet in thickness in the 4000-foot distance between the well and the river, assuming a porosity of only 0.20, yields a figure of 1.2 million gallons for each strip perpendicular to the river and one foot wide. Although there is no reason to believe that water is not coming toward the well from all directions equally, if we assume that all the water is coming from the river through a strip 300 feet wide,

1. Cons. Engr., Stuttgart, Ark.

there is stored in that strip some 360 million gallons of water, most of which must first be pumped out before river water which has infiltrated can get to the well. It is unlikely that this could be accomplished in 12 days (as indicated on Figure 12) or even in 100 days, assuming a discharge of 2,500 gpm from the irrigation well. Moreover, once a volume of aquifer became salty, it would take a good deal of pumping to remove the salt—the drop in salinity would be slow and not at all abrupt, as shown in Figure 12.

2) None of the hydraulic data given in the paper even imply that the Vermilion River is a source of aquifer recharge.

An alternate explanation of the data presented in Figure 12 is this:

The rice farmers in the area are cost conscious and salinity conscious: it is advantageous to use surface water rather than ground water whenever possible both from the standpoint of power-cost and from the standpoint of ease and flexibility of the rice-farming operation. When the salt content of the Vermilion River rises, the farmers stop using it and turn on their well pumps. When the salt content of the river goes down to the normal salinity of the ground water, the farmers shut off the well pumps and resume irrigation with river water.

If it is assumed that the increase in the salt content in the well water (Figure 12) is due to local coning of salt water, the probable time lag would agree with the data presented. There would be a definite relaticnship between river water salinity and increase in well water salinity: the first would lead to the second. However, it would not be necessary to believe that water from the Vermilion River moves 4000 feet through the aquifer in 12 days to enter the well. The actual sequence of events might be:

1) Chloride content of Vermilion River water rises.
2) Farmer shuts down river intake.
3) Farmer starts to pump irrigation well.
4) Coning of salt water underlying the well causes chloride content of well water to rise.
5) Chloride content of Vermilion River water goes down due to increased runoff from drainage area due to precipitation.
6) Farmer shuts off well pump. Salt water cone under well disappears.
7) Farmer pumps water from river intake.
8) Cycle resumes with item 1.

In the above sequence the word "farmer" is used to mean "most of the irrigators in the area."

Such a sequence of events eliminates the need for aquifer recharge from the Vermilion River; it does not contradict the observed hydraulic data which indicate that such recharge does not occur; it explains the observed rapid variation in chloride content in water from well VE 75; and it does not conflict with any of the other data presented by the author. If this explanation is accepted, the author's conclusions stand in need of revision.

With respect to the area which may be served by surface water taken from the Vermilion River: the elimination of salt water from the river by means of a barrier will result in the substitution of surface water for ground water in the irrigation of rice and will reduce the contamination of the aquifer by coning. In fact, since ground water will not be needed for irrigation it will be conserved for domestic, municipal, and industrial purposes. However, the coning of the deeper-lying salt water will continue to be a present or potential danger in areas which cannot replace the ground-water supply with surface water.

In other areas of southwestern Louisiana supplied by the same aquifer and not served by water from the Vermilion River, the construction of a salt water barrier in the Vermilion River will have no direct or indirect effect on the continued threat of salt water contamination of the aquifer.

PROCEEDINGS-SEPARATES

The technical papers published in the past year are presented below. Technical-division sponsorship is indicated by an abbreviation at the end of each Separate Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways (WW) divisions. For titles and order coupons, refer to the appropriate issue of "Civil Engineering" or write for a cumulative price list.

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MARCH: 414(WW)[d], 415(SU)[d], 416(SM)[d], 417(SM)[d], 418(AT)[d], 419(SA)[d], 420(SA)[d], 421(AT)[d], 422(SA)[d], 423(CP)[d], 424(AT)[d], 425(SM)[d], 426(IR)[d], 427(WW)[d].

APRIL: 428(HY)[c], 429(EM)[c], 430(ST), 431(HY), 432(HY), 433(HY), 434(ST).

MAY: 435(SM), 436(CP)[c], 437(HY)[c], 438(HY), 439(HY), 440(ST), 441(ST), 442(SA), 443(SA).

JUNE: 444(SM)[e], 445(SM)[e], 446(ST)[e], 447(ST)[e], 448(ST)[e], 449(ST)[e], 450(ST)[e], 451(ST)[e], 452(SA)[e], 453(SA)[e], 454(SA)[e], 455(SA)[e], 456(SM)[e].

JULY: 457(AT), 458(AT), 459(AT)[c], 460(IR), 461(IR), 462(IR), 463(IR)[c], 464(PO), 465(PO)[c].

AUGUST: 466(HY), 467(HY), 468(ST), 469(ST), 470(ST), 471(SA), 472(SA), 473(SA), 474(SA), 475(SM), 476(SM), 477(SM), 478(SM)[c], 479(HY)[c], 480(ST)[c], 481(SA)[c], 482(HY), 483(HY).

SEPTEMBER: 484(ST), 485(ST), 486(ST), 487(CP)[c], 488(ST)[c], 489(HY), 490(HY), 491(HY)[c], 492(SA), 493(SA), 494(SA), 495(SA), 496(SA), 497(SA), 498(SA), 499(HW), 500(HW), 501(HW)[c], 502(WW), 503(WW), 504(WW)[c], 505(CO), 506(CO)[c], 507(CP), 508(CP), 509(CP), 510(CP), 511(CP).

OCTOBER: 512(SM), 513(SM), 514(SM), 515(SM), 516(SM), 517(PO), 518(SM)[c], 519(IR), 520(IR), 521(IR), 522(IR)[c], 523(AT)[c], 524(SU), 525(SU)[c], 526(EM), 527(EM), 528(EM), 529(EM), 530(EM)[c], 531(EM), 532(EM)[c], 533(PO).

NOVEMBER: 534(HY), 535(HY), 536(HY), 537(HY), 538(HY)[c], 539(ST), 540(ST), 541(ST), 542(ST), 543(ST), 544(ST), 545(SA), 546(SA), 547(SA), 548(SM), 549(SM), 550(SM), 551(SM), 552(SA), 553(SM)[c], 554(SA), 555(SA), 556(SA), 557(SA).

DECEMBER: 558(ST), 559(ST), 560(ST), 561(ST), 562(ST), 563(ST)[c], 564(HY), 565(HY), 566(HY), 567(HY), 568(HY)[c], 569(SM), 570(SM), 571(SM), 572(SM)[c], 573(SM)[c], 574(SU), 575(SU), 576(SU), 577(SU), 578(HY), 579(ST), 580(SU), 581(SU), 582(Index).

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FEBRUARY: 608(WW), 609(WW), 610(WW), 611(WW), 612(WW), 613(WW), 614(WW), 615(WW), 616(WW), 617(IR), 618(IR), 619(IR), 620(IR), 621(IR)[c], 622(IR), 623(IR), 624(HY)[c], 625(HY), 626(HY), 627(HY), 628(HY), 629(HY), 630(HY), 631(HY), 632(CO), 633(CO).

c. Discussion of several papers, grouped by Divisions.
d. Presented at the Atlanta (Ga.) Convention of the Society in February, 1954.
e. Presented at the Atlantic City (N.J.) Convention in June, 1954.